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ABSTRACT

This paper presents a case history of a deep excavation for building foundation in Shanghai soft deposit. The excavated area was about 15916 m^2. The excavation depth at the tower part was 15.0 m and at the podium part was 14.5 m to 15.2 m. Around the excavated pit, there existed many facilities including roads, the shield tunnels of the No.7 Metro, buildings, and pipelines. Zoned excavation method was adopted in this project. The excavation was divided into a relatively small pit (Zone I) and a big pit (Zone II). The small pit was just adjacent to the tunnels. The small pit and the big pit were separated by temporary diaphragm walls. The small pit was firstly constructed by bottom-up method. The big pit was constructed after the completion of the construction of the small pit. Top-down method was adopted for the big pit. Detailed construction sequences were introduced. An extensive instrumentation program was carried out to monitor the performance of the excavation and the adjacent facilities. The monitored data of the retaining structure and the surrounding facilities were analyzed. Observed results show that zoned excavation method was effective in controlling the deformation of the adjacent tunnels.

PROJECT DESCRIPTION

The Jing'an Transportation Junction & Retail Complex Project located in the downtown area of Jing'An district in Shanghai. The project comprised a 20-story tower and a 8-story podium with their three levels basements connected together. The excavation site occupied an area of about 15916 m^2. It was bounded by Changde Road on the east side, Jiaozhou Road on the west side, Zhaojiaqiao Road on the north side, and Yuyuan Road on the south side, as shown in Fig. 1. The shape of the excavation site was slightly regular (approximately 137 m×116 m) except the southern side was changed by an existing building. The excavation depth at the tower part was 15.0 m and at the podium part was 14.5 m to 15.2 m.

The environment was quite complex around the excavation. The shield tunnels of the No.7 Metro (in operation) were laid about 12.0 m under the Changde Road. The tunnels were approximately parallel to the east side of the excavation and the minimum distance between the tunnels and the excavation was only about 8.6 m. On the south side, the existing old Building B was about 12.2 m away from the excavation. Pile raft foundation was adopted for the 8-story Building B and the pile length was about 18 m. There were also many existing buildings on the west and north side of the excavation and the minimum distance of these buildings to the excavation was about 10.2 m. Moreover, there were many pipelines such as power cables, water supply pipes, and gas pipelines were distributed around the foundation pit. The minimum distance between the pipelines and the excavation was about 1.7 m.

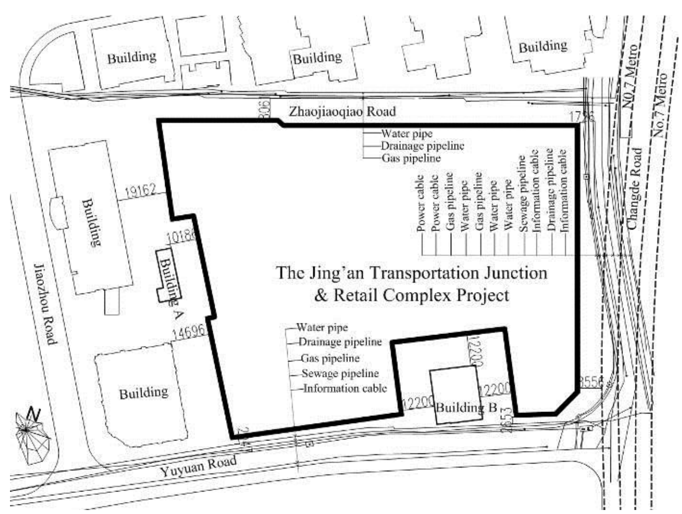
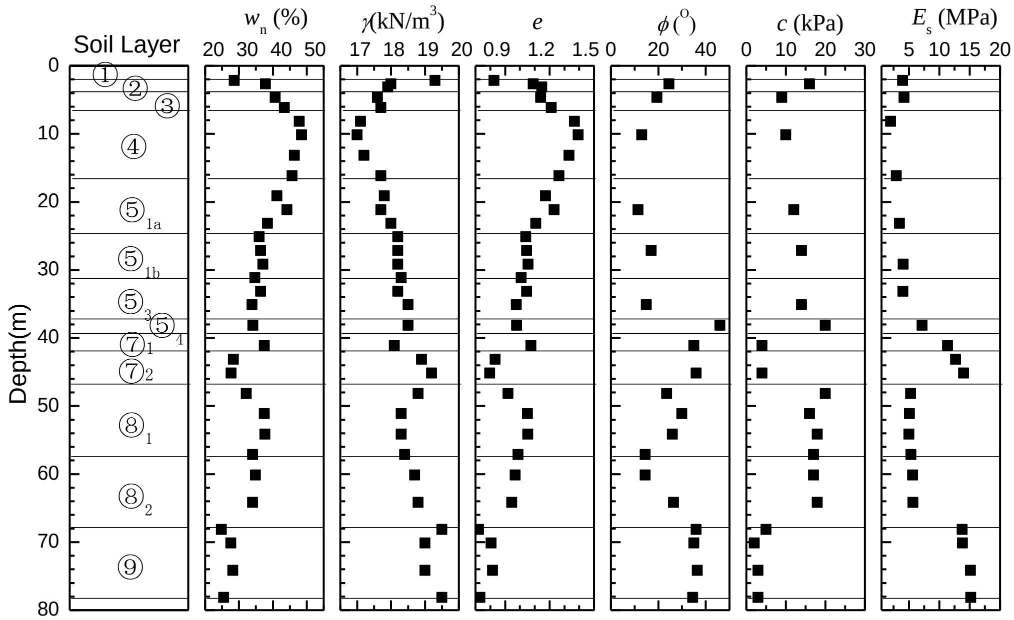


Fig.1. Plane view of the excavation site.



Note: γ=unit weight; w_n=natural water content; e=voids ratio; c=cohesion obtained from direct shear test; φ=angle of internal friction obtained from direct shear test; $E_{s0.1\sim0.2}$=compressibility modulus

Fig.2. Soil profile and geotechnical parameters at the construction site.

GROUND CONDITION

Shanghai is located in the front fringe of the Yangtze River Delta and washed by the East China Sea on the east and Hangzhou Bay on the south. According to the geotechnical investigation report (SSGEC, 2008), the ground soils at the construction site were mainly thick soft soils comprising Quaternary alluvial and marine deposits. As shown in Fig. 2, from ground surface to a depth of about 80 m, the underground could be divided into 8 layers, among which Layer ⑤, Layer ⑦ and Layer ⑧ could be subdivided into 4, 2 and 2 sub-horizontal layers, respectively.

The first layer (Layer ①) was a less than 2.0-m-thick artificial fill in general. The second layer (Layer ②) was brownish yellow silty clay with medium-soft plastic. The third and fourth layers were very soft silty clay (Layer ③) and very soft clay (Layer ④). This two layers had large void ratio, low shear strength and high compressibility. Thickness of Layer ④ was about 10 m. Mean value of the water content of Layer ④ was about 45% and undrained shear strength was about 35 kPa. It was the main layer affecting the excavation behavior. Underlying was the fifth layer, which was divided into four sub-layers, namely, Layer ⑤$_{1a}$, Layer⑤$_{1b}$, Layer⑤$_3$, and Layer ⑤$_4$).The fifth layer was mainly grayish silty clay with medium palstic and medium to high compressibility. The physical and mechanical properties of this layer were much better than that of Layer ④. The next layer was divided into two sub-layers, namely, Layer ⑦$_1$ and Layer ⑦$_2$. Layer ⑦$_1$ was medium to slightly dense sandy silt and Layer ⑦$_2$ was silty sand. The SPT N values of Layer ⑦$_1$ and Layer ⑦$_2$ were about 35 and 41, respectively. Silt and sand in Layer ⑦$_1$ and Layer ⑦$_2$ composed the first confined aquifer in Shanghai. Underlying was about 20 m thick silty clay which was divided into two sub-layers, namely, Layer ⑧$_1$ and Layer ⑧$_2$, with medium compressibility. Underlying was the fine silty sand (Layer ⑨) with SPT N value of about 66.

DESIGN OF THE SUPPORTING SYSTEM OF THE EXCAVATION

Considering the excavation area, the excavation depth, and the protection requirements of the adjacent facilities, zoned excavation method was adopted in this project. The excavation was divided into a relatively big pit (Zone Ⅱ) and a small pit (Zone Ⅰ), as shown in Fig. 3. Zone Ⅰ was just adjacent to the tunnels of the No.7 Metro. Zone Ⅰ and Zone Ⅱ were separated by temporary diaphragm walls. Zone Ⅰ was firstly constructed by bottom-up method. Zone Ⅱ was constructed after the completion of the construction of Zone I. Top-down method was adopted for Zone Ⅱ. As Zone I was quite small, deformation of this part might be well controlled so that the effects of the excavation on the adjacent No.7 Metro tunnels could be reduced.

The excavation of Zone Ⅰ was retained by diaphragm wall. The thickness of the diaphragm wall adjacent to the No.7 Metro tunnels was 1000 mm. The embedded length (i.e., length under the bottom of the excavation) of this diaphragm wall was 22.9 m. The thickness of the temporary diaphragm wall used to separating Zone I and Zone II was 800 mm and its embedded length was 15.3 m. The diaphragm walls were supported by four levels of struts. The first level struts were 800 mm × 800 mm RC struts with a horizontal spacing of 6.0 m. The three levels struts beneath the first level struts were steel tube struts with diameter of 609 mm and thickness of 16 mm. The horizontal spacing of the steel tube struts was 3.0 m. Fig. 4 shows the sectional view of the supporting system at Zone I.

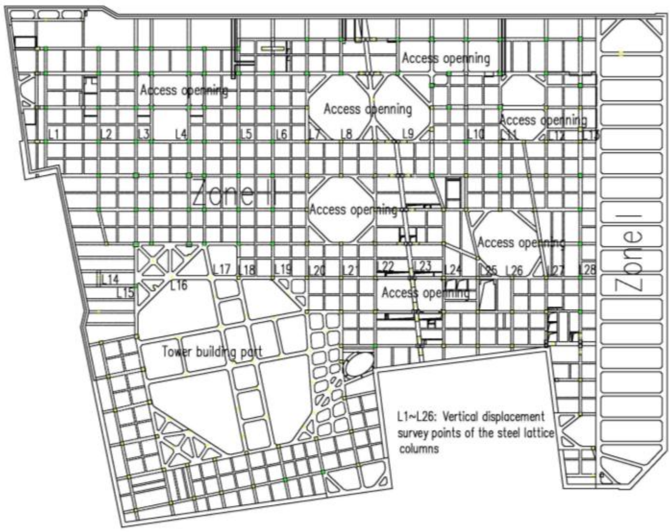


Fig.3. Plane view of the zoned excavation.

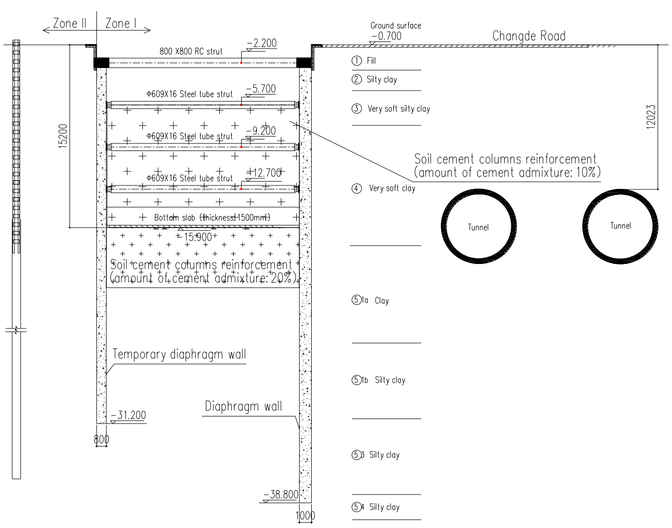


Fig. 4. Sectional view of the supporting system at Zone I.

The excavation of Zone II was also retained by diaphragm wall. The thickness of the diaphragm wall was 800 mm. The embedded length was 17.0 m. The diaphragm wall was braced at the floor levels by the three basement slabs. The B0 slab was also used as platform for soil excavators, dump trucks and other construction machines. Big access openings (see Fig. 3) were distributed in the slabs to facilitate the removal of the excavated materials and delivery of building materials. Big access opening was also set at the tower building part. The excavation at this part was temporarily supported by RC struts. After the final excavation, the tower building would be constructed by bottom-up method. Fig. 5 shows the sectional view of the supporting system at Zone II.

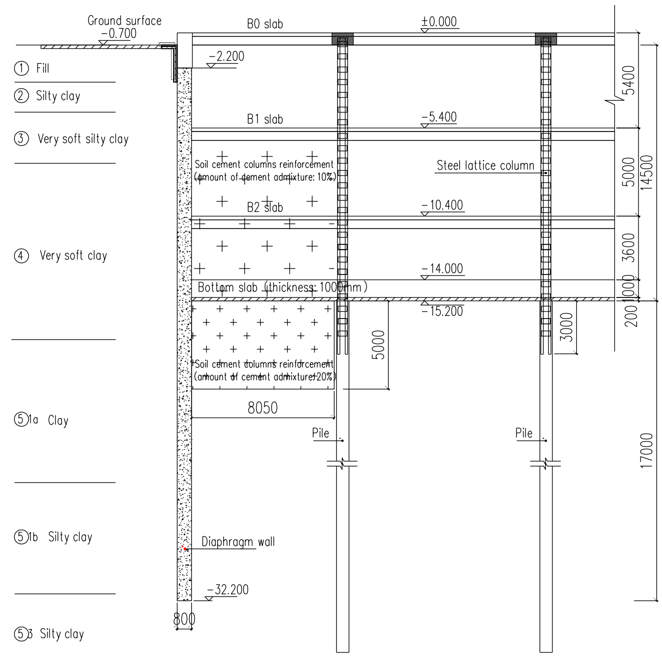


Fig. 5. Sectional view of the supporting system at Zone II.

In order to further reduce deformation of the walls and the effects of the excavation on the adjacent facilities, soils in the passive area of Zone I and local areas of Zone II were reinforced by soil cement columns which were formed by triple shaft mixing machines. The amount of cement admixture above and beneath the bottom of the excavation was 10% and 20%, respectively.

CONSTRUCTION CONSEQUENCE AND MONITORING

Construction of the excavation involved diaphragm wall and vertical supports construction, soil cut, horizontal struts construction and underground slabs construction. The diaphragm walls, piles, and soil reinforcement were firstly constructed, then excavation of Zone I commenced using bottom-up method. During the construction of the underground structure of Zone I, excavation of the first layer soil and construction of the B0 slab in Zone II started. The slabs of Zone I and Zone II were connected layer by layer with the processing of construction of Zone II using top-down method. The temporary separating diaphragm wall was accordingly demolished step by step. Table 1 shows details of the whole construction procedure. Fig. 6 gives a photo of the construction site showing that the construction of Zone I was finished and the third excavation of Zone II was conducting.

Table 1. Excavation Sequences

Stage	Interval (d)	Construction activities	
1	211	Construction diaphragm walls, piles, and soil reinforcement	
2	4	Zone I	Excavate to elevation of -2.6 m and cast the 1st RC strut
3	5		Excavate to elevation of -6.1 m and install the 2nd steel tube struts
4	8		Excavate to elevation of -9.6 m and install the 3rd steel tube struts
5	10		Excavate to elevation of -13.1 m and install the 4th steel tube struts
6	8		Excavate to elevation of -15.9 m and cast the cushion
7	10		Cast the bottom slab
8	54	Zone I and Zone II	Demolish the struts and construct the underground structure of Zone I, at the same time Zone II excavate to elevation of -2.2 m and cast B0 slab
9	47	Zone II	Excavate to elevation of -6.1 m and cast B1 slab
10	86		Excavate to elevation of -11.1 m and cast B2 slab
11	66		Excavate to the final elevation and cast the bottom slab

Fig. 6. Photo of the construction site showing the cutting of the third layer soils in Zone II.

Field monitoring is an important mean of providing feedback to designers during excavation and of verifying design assumptions to reduce risk during the excavation. Observed performance of deep excavations has been reported by many researchers (such as O'Rourke 1981; Clough and O'Rourke 1990; Ng 1998; Ou 1998; Finno and Bryson 2002; Blackburn and Finno 2007). These have provided the opportunities to the engineers to understand the characteristics of wall deformation and ground movements. To monitor the performance of the

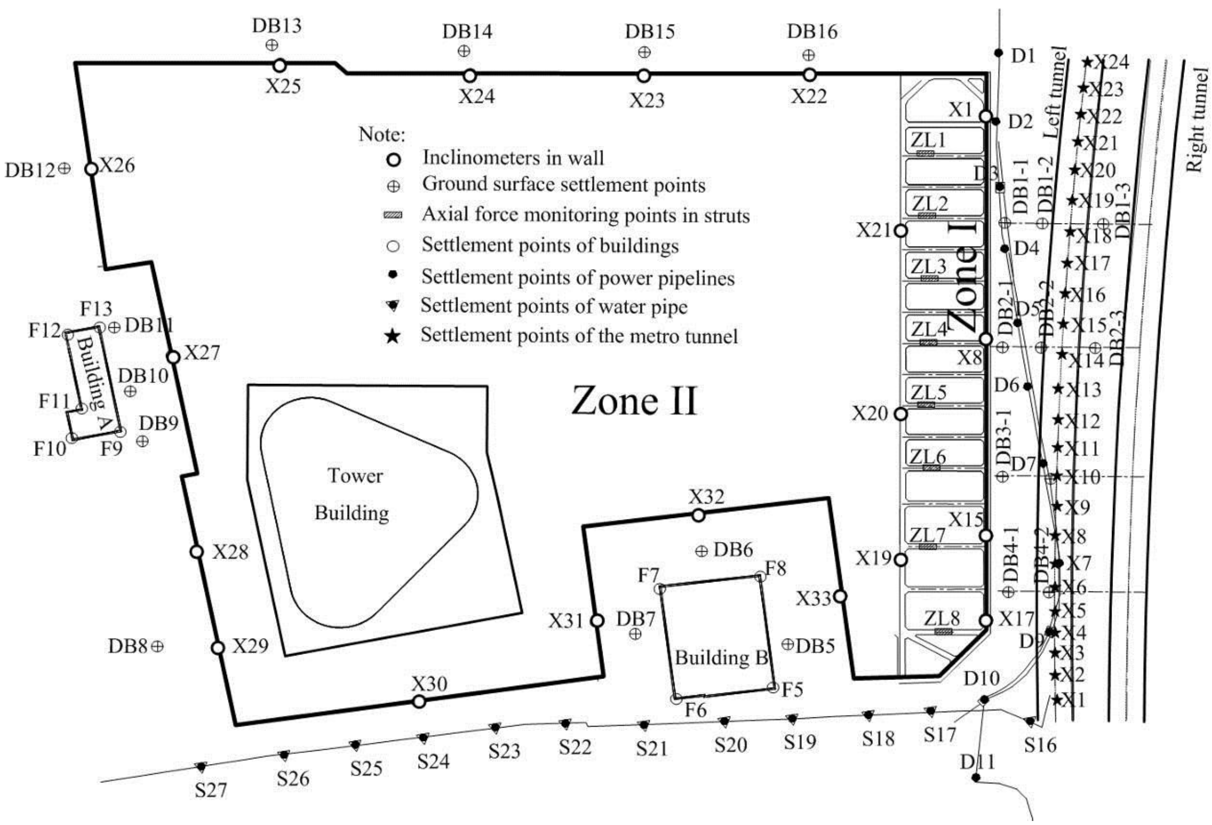


Fig.7. Layout of instrumentation at the construction site.

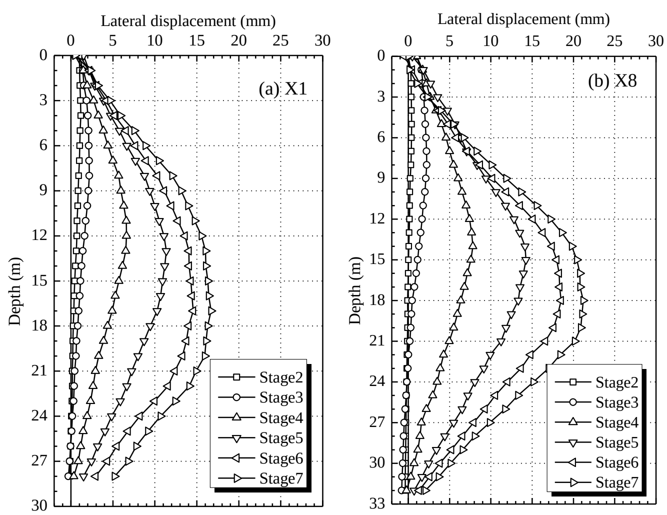
excavation and the effects of the excavation on the surrounding facilities, various instruments were installed at the construction site (see Fig. 7). Inclinometer tubes with length equal to the diaphragm wall were installed in the wall to measure the lateral displacement of wall. Vibrating steel bar stress gauges were used to measure the axial forces of struts. Displacement survey points were installed to monitor the settlements of ground surface, tunnels, buildings and the pipelines.

MONITORED RESULTS AND ANALYSIS

Lateral displacement of diaphragm wall

Fig. 8 shows lateral displacement curves of diaphragm wall at three different locations in Zone I (X1 near the north corner, X8 near the middle of the wall, and X17 near the south corner of the site, see Fig. 7). It can be seen that the lateral displacement of wall gradually developed into bulging profiles as the excavation proceeded. The most obvious deflection increments were observed in Stage 4, Stage 5 and Stage 6. The maximum wall deflection δ_{hm} of X1, X8, and X17 at stage 7 were 16.7 mm, 21.2 mm, and 16.9 mm, respectively. Lateral displacement at X1 and X17 was much smaller than that at X8 due to the corner effect (Lee et al., 1998). It took 10 days to cast the bottom slab. During this time, all of the inclinometers had noticeable lateral displacement increment as shown in Fig. 8. This might be caused by the creep property of the soft soil as well as consolidation. It can also be seen that the maximum lateral displacement generally occurred near the elevation of the bottom of the reinforced soil under the bottom of the excavation.

Fig. 9 shows typical lateral displacement curves of diaphragm wall at X30 and X33 in Zone II. The wall also gradually developed into bulging profiles as the excavation proceeded. The maximum wall deflection δ_{hm} of X30 and X33 at stage 11 were 70.2 mm and 60.5 mm, respectively. The deformation of walls in Zone II was much larger than that in Zone I. This



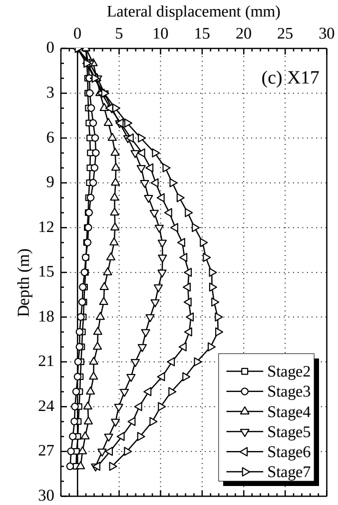


Fig.8. Lateral displacement of diaphragm wall at different stages in Zone I.

was attributed to the following facts: (1) The area of Zone I was much smaller than that of Zone II so that excavation of Zone I was much faster than that of Zone II; (2) Average vertical space of the struts in Zone I was much smaller that that in Zone II; (3) Wall thickness of Zone I was much larger than that of Zone II (except the temporary separating wall); (4) All the soils in the passive area (from the elevation of the second strut to the elevation of 5 m beneath the bottom of the excavation) of Zone I were reinforced by soil cement columns while only local areas of Zone II were reinforced. It can be seen that the maximum lateral displacement generally occurred near the bottom of the excavation.

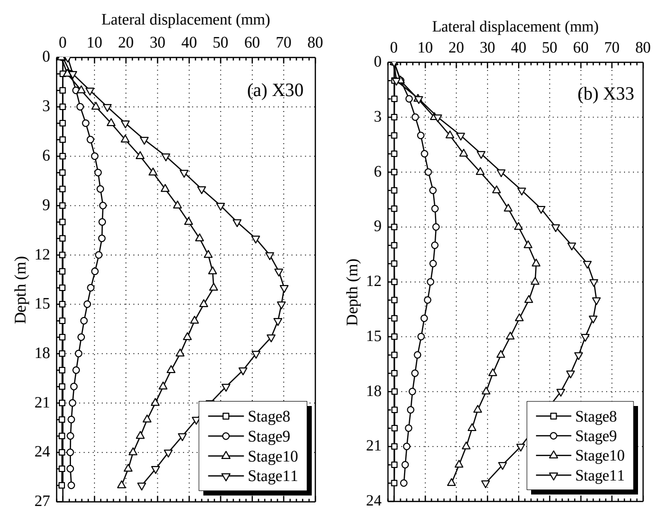


Fig. 9. Lateral displacement of diaphragm wall at different stages in Zone II.

The values of the ratio of the maximum lateral displacement of wall (δ_{hm}) to the excavation depth (H) of Zone I and Zone II were 0.14% and 0.48%, respectively. Xu et al. (2008) collected 93 case histories of diaphragm wall deformation due to deep excavations in Shanghai soft deposits. In their study the maximum lateral displacement of wall ranges from 0.1%*H* to 1.0%*H*, with a mean value of about 0.42%*H*. The measured results in Zone I fell in the lower bound of the ranges that collected by Xu et al. (2008) because of the deformation control measures mentioned above. Study conducted by Wang et al. (2010) shows that value of maximum deformation of 32 excavations constructed by top-down method in Shanghai ranges from 0.1%*H* to 0.55%*H*, with an average value of 0.27%*H*. The measured results in Zone II generally fell in the range proposed by Wang et al. (2010).

Ground surface settlement

The maximum ground surface settlement near the small excavation Zone I and the big excavation Zone II were 16.1 mm (occurred at point DB3-2) and 136.2 mm (occurred at point DB7), respectively. Fig. 10 shows the relationship between d/H and δ_v/H, where d is the distance from a ground surface point to the excavation, δ_v is the settlement of a ground surface point. Regions of the distribution of ground surface settlement proposed by Peck (1969) and upper bound ground surface settlement proposed by Wang et al. (2010) are also displayed in Fig. 10 for comparison. It can be seen that ground surface near the small excavation (Zone I) was much smaller than that near the big excavation (Zone II). The unusual large settlements at point DB5 and DB7 will be mentioned in the following. All of the ground surface settlement points (except point DB5 and DB7) fall within zone I and the magnitude of the measured ground surface settlement is substantially far smaller than those observed in similar ground conditions by Peck (1969). All the ground surface settlement points (except point DB7) fall within the upper bound proposed by Wang et al. (2010).

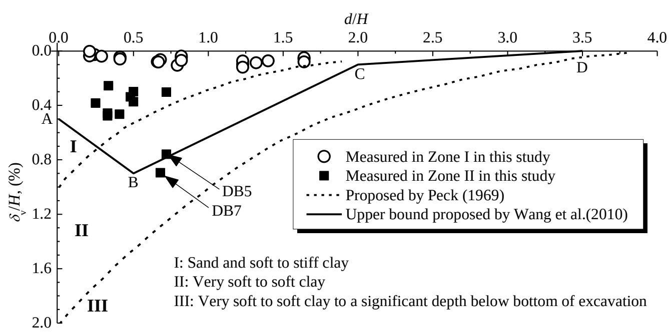


Fig. 10. Distribution of ground surface settlement.

Axial Forces in the struts

Fig. 11 shows the axial forces in the four levels struts in Zone I. Axial force in the second and third level struts was the largest among the four level struts while axial force in the first level strut was the smallest. The maximum axial force in the first level strut was 683 kN at ZL3-1, whereas in the second level strut was 1633 kN at ZL4-2, in the third level strut was 1880 kN at ZL4-3, and in the fourth level strut was 1411 kN at ZL6-3. The axial force in the first struts increased during the second excavation, after that it nearly remained unchanged. The axial force in the second, third and fourth levels struts increased obviously during the next excavation. After that a small amplitude growth was observed.

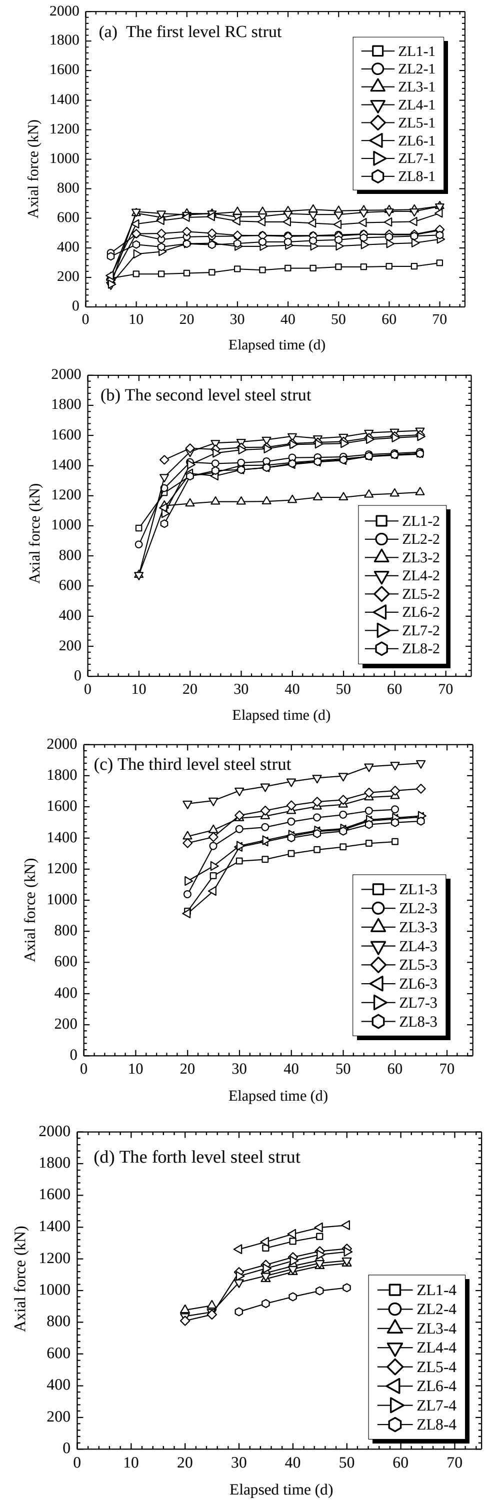


Fig. 11. Axial forces in the four levels struts in Zone I.

Displacement of the steel lattice columns

Fig. 12 depicts the vertical displacement of the steel lattice columns in Zone II constructed by top-down method. It can be seen that most of the steel lattice columns settled during the construction of the B0 slab. However, the value of settlement was quite small. After that all the steel lattice columns were gradually uplifted due to unloading of the soil. The maximum vertical displacement was 45.8 mm occurring at L21. The vertical displacement at Z1, Z2, Z13, Z14, Z15, and Z28 (see Fig. 3) was much smaller than that at the other points. This is because that Z1, Z2, Z13, Z14, Z15, and Z28 were near the wall while the other points located at the center of the pit.

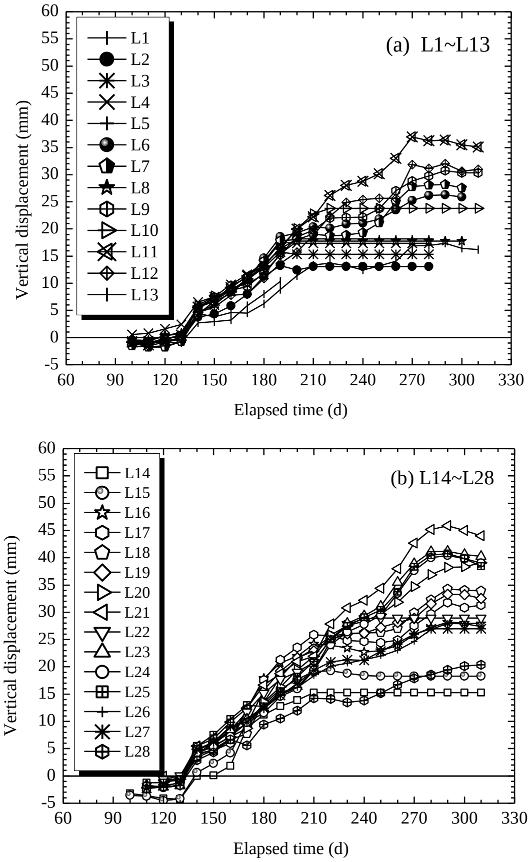


Fig. 12. Vertical displacement of the steel lattice columns in Zone II.

Vertical displacement of the metro tunnel

Fig. 13 shows the vertical displacement of the left metro tunnel at Stage 7 and Stage 11. It can be seen that when the excavation of Zone I finished (Stage7), settlement was observed at most of the monitoring points except uplift was observed at a few points of the north side. The maximum settlement and uplift were 8.5 mm and 2.8 mm, respectively. During the construction of Zone II, further settlements at all the monitoring points were observed. The maximum settlement of the tunnel at Stage11 was 13.1 mm. This value was less than the deformation controlling criteria in Shanghai (20 mm) and thus the normal operation of the metro was no affected.

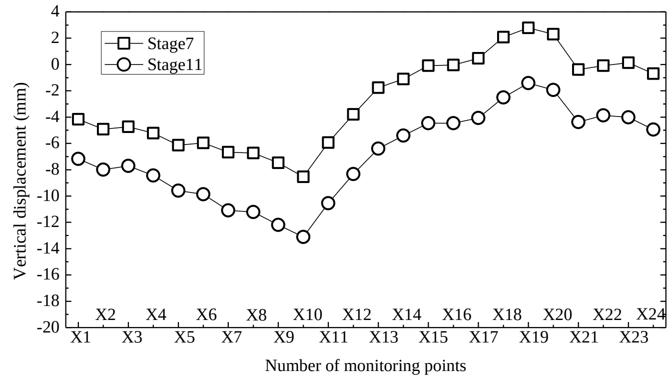


Fig. 13. Vertical displacement of the metro tunnel.

Settlement of buildings and pipelines

Fig. 14 shows the vertical displacements of the surrounding buildings versus time. Building A was a 2-storey substation with shallow foundation. Fig. 14 (a) shows that Building A settled with the processing of excavation of Zone II. The maximum settlement of Building A was 104.6 mm. Though the settlement of the building was a little large, the deferential settlement was insignificant. Thus the normal use of this building was not affected. Fig. 14(b) shows that Building B was subjected to a very large settlement during the excavation of Zone II. The maximum settlement was 172 mm. The maximum angular distortion of Building B was 1/270. According to Skempton and Macdonald (1956), angular distortions of about 1/300 corresponded to cracking in panel walls. Building B was a RC frame structure. A few cracks were observed in the east, north, and west outside walls of Building B at Stage 10. However, no structural damage was observed in Building B. Building B and the nearby ground surface subjected to such a large settlement might be due to the following reasons: (1) Three sides of the building faced the excavation. Additive settlement effect caused the building settling more. (2) The 8-storey building itself was overload acting beside the excavation. This may cause more deformation of the soils. (3) Though piles were adopted for this building, the pile length was only 18 m and the pile toes were embedded into the relatively weak 5a clay. Thus the piles did not play a good supporting role for the building. (4) A hidden creek was found under the building during the excavation. This made the foundation soil of the building even worse. (5) A detailed layered and blocked excavation plan was initially required near the building. However, the construction contractor did not follow the origin plan. The soil around the building was excavated at one time and the construction of the slabs lasted quite a long time. The building might settle more due to the creep property of the soft soil.

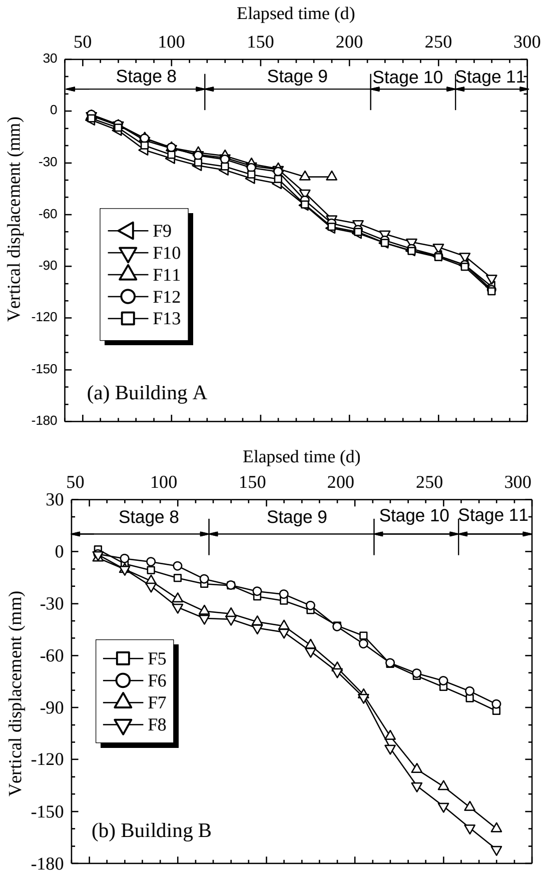
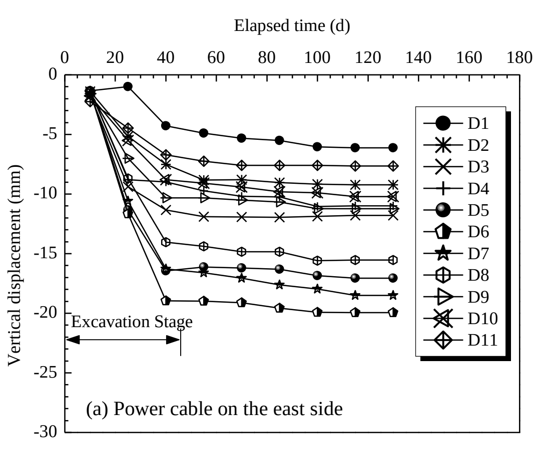


Fig.14. Settlements of Buildings versus time.

Fig. 15(a) shows the settlement of the power cable on the east side of the excavation (near Zone I). It can be seen that the settlement mainly caused during the excavation stage of Zone I. After that, the settlement became stable. As the lateral displacement of the diaphragm wall on this side was quite small, the maximum settlement of the power cable was only about 20.0 mm. Fig. 15(b) shows the settlement of the water pipe on the south side of the excavation (near Zone II). The maximum settlement of the water pipe was 81 mm. Though the settlement of the water pipe was quite large, its normal use was not affected.



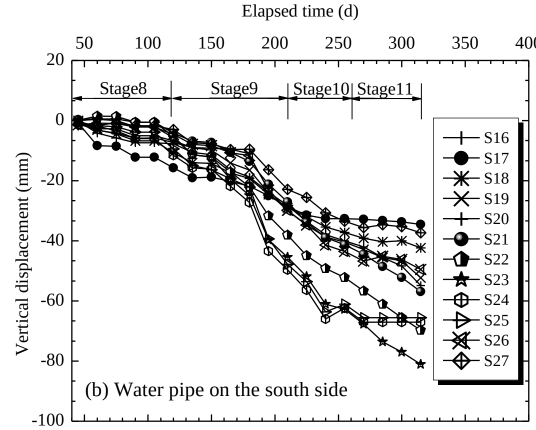


Fig.15. Settlement of the surrounding pipelines.

SUMMARY AND CONCLUSION

The Jing'an Transportation Junction & Retail Complex Project located in congested urban area with quite poor geological condition and strict requirements of environmental protection. Zoned excavation method was adopted in this project. The excavation was divided into a relatively small pit (Zone I) and a big pit (Zone II). The small pit was firstly constructed by bottom-up method and the big pit was constructed using top-down method after the completion of the construction of the small pit. Monitored results show that the zoned excavation method was effective in controlling the deformation of the adjacent tunnels. The maximum settlement of the tunnels was only 13.1 mm and the normal operation of the metro was not affected. Deformation of the small pit was quite smaller than that of the bit pit. The adjacent environment was well protected except that the settlement of Building B was quite large due to some factors such as three sides of the building facing the excavation, overload caused by the building itself, a hidden creek under the building making the foundation soil of the building even worse, and the construction contractor's unreasonable excavation. Through a few cracks appeared in the outside walls of the building, no structural damage was observed in Building B.

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